INSTITUTE of HYDROLOGY

WEST BAY GROUNDWATER MODELLING STUDY, DOHA

Interim Report

INSTITUTE of HYDROLOGY

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West Bay Groundwater Modelling Study, Doha.

Interim Report

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LIST OF CONTENTS

List of Tables

List of Figures

Summary

The report describes the results of data analysis completed to date.

The following preliminary conclusions can be drawn from the information collected:

* The potential flooding problems facing the West Bay area are:

- a short term problem resulting from high intensity, short duration rainstorms. These cause flooding of low-lying depressions with inadequate surface drainage, and in areas with shallow water levels and low permeability surface deposits.

- a longer term and more serious threat from increased recharge as urban development continues. Recharge from leaking water pipes and garden irrigation will raise water levels in much the same way as has already happened in central Doha.

* The geology can be considered in terms of three main lithological units: hydraulic fill, original coastal sediments, Dammam dolomite. Regionally there appears to be hydraulic continuity between these formations. The impermeable base to the sequence is the Midra Shale, which lies immediately below the dolomite.

* Areas having the shallowest water tables are not, as might be expected, near to the coast, but in a belt of land extending parallel to and just inland of the former coastline. These are associated with a topographic low in this region. Backfilling has built up the general elevation of the peninsula to 3m QND, but inland of the original shoreline the ground level has not been raised to the same extent and groundwater levels are within 2m of the surface.

* Analysis of grain size data, pumping and slug tests indicates that the geometric mean hydraulic conductivity for coast and hydraulic fill material to be 25 and 35 m/d, respectively. A preliminary estimate of the aquifer storage coefficient for both types of material based on tidal response is 11%. However, more analysis and data is needed to confirm these estimates.

Hydraulic properties for the dolomite will be based on information given in the ASCO (1983) and JICA (1987) reports.

* The model to be used for drainage design is a 3-D finite element code called FLAMINCO. There are 9 layers each of 161 nodes and 8 layers of 280 elements: a total of 1449 nodes and 2240 elements. Care has been taken with the model grid to ensure that element boundaries follow the lines of major roads, since it is along roads that drains are most easily installed. This reduces the problems of transferring a drainage network based on the model grid to the field.

A limited amount of data collection and some further analysis is necessary before predictive modelling can begin. It is anticipated that the final report will be submitted in April 1990.

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Chapter 1

BACKGROUND

1.1 INTRODUCTION

The West Bay district of Doha is formed from reclaimed land along the northern shore of Doha Bay. This area is generally below an elevation of 3m Qatar National Datum (QND) and has shallow groundwater levels which are locally within 1m of the ground surface.

Some of the lower lying parts of the district are subject to occasional flooding after heavy rainfall when the water table rises to the surface and surface run-off collects in closed depressions. These floods may persist for several days but occur infrequently, the last occasion being in February 1988.

Urban development of the West Bay area is now taking place. There is growing concern that the resulting increase in water demands will lead to an increase in recharge from garden irrigation and leaking pipes. This could cause a permanent rise in the water table and an increase in the frequency and duration of flooding unless water level control measures are implemented. Such flooding problems already occur in other parts of Doha.

It is proposed to install a network of groundwater drains to combat the expected rise in groundwater levels and thereby prevent future long term flooding.

The design of a drainage network requires an understanding of the local hydrogeology, especially in regard to the hydraulic characteristics of the complex sequence of deposits forming the West Bay area. Prior to this study there was limited information on the newly created, reclaimed area and, consequently, a programme of data collection has been undertaken during 1989 to provide such information.

A preliminary analysis of the new information is presented in this interim report. The main objective of the study is to use this information to construct a numerical groundwater model of the West Bay region to predict those areas at greatest risk from flooding and to assist the design of a drainage network to alleviate such flooding.

1.2 STUDY AREA

The area of investigation is about 25 km2. It includes the entire district of West Bay and extends 6 km inland, as shown in Figure 1.1.

The development of West bay is at an early stage. Figure 1.2 shows the existing extent of the urban development. The main areas of present development are the two large housing estates located just inland of the former coastline. On the penninsula itself there are offices and an hotel along the southern shore and embassy buildings along the eastern and northern coasts. Much of the central part of the area has yet to be developed, although a number of major urban development projects are planned.

Reclamation of the West Bay area began in the late 1970's and continued into the early 1980's. Backfilling is still taking place which makes it difficult to obtain a reliable ground elevation map. Figure 1.1 shows the best estimate that can be made of the topography and has been compiled from maps prepared in 1983 at a scale of 1:2000 supplemented by elevation data obtained from the levelling of new boreholes drilled during this study. Particular topographic features indicated by this map include:

- a marked increase of slope inland from the 4m QND contour, which coincides with the position where the dolomite bedrock rises from beneath the cover of coastal deposits and fill material

- the generally low elevation (less than 3m QND) of most of the penninsula and much of the original foreshore area

- a 5m high ridge parallel to the coast along the eastern and northern perimeters of the penninsula

- the position of the former shoreline shown as a dotted line (this is based on a pre-reclamation map of 1964).

The area of most concern is that lying seward of the 4m QND contour this being where a future rise in water levels would have the greatest impact.
The materials used for the reclaimation of the West Bay area consist of "hydraulic fill" and "desert fill". The hydraulic fill is formed of sediments dredged from near shore and pumped as a slurry into a bunded area to raise the level of the sea floor above sea level. Desert fill, which is still being placed at the present day, consists of rubble brought from inland. This is commonly mixed with other material, such as plastic, wood or metal, and is used as a final stage of backfilling. These deposits are still subject to compaction.

The fill materials have been placed over a complex sequence of coastal and near-shore silts, sands and gravels, which in turn rest on dolomitic limestone.

1.3 SCOPE AND FORMAT OF REPORT

This report presents the interim results of a hydrogeological investigation carried out to provide data for a numerical model of the West Bay area. It is intended to show how the information obtained from the recent field investigations has greatly improved the previously limited knowledge concerning the hydrogeological conditions of the West Bay and the way in which the complex conditions can be simplified to enable an appropriate numerical model representation.

Analysis of existing information is ongoing. Further information, such as routine water level data, is still being collected. Values for the different parameters required by the model, such as hydraulic conductivity and storage coefficient, are discussed, although further analysis is required to refine these values for the final model.

Whilst the report includes a brief description of the model, it does not include any predictions. These, together with a more detailed analysis of the hydrogeological data, will be given in a Final Report to be presented in April 1990.

The data collection programme undertaken during 1989 is summarised in Chapter 2. The borehole data have been used to describe the geology of the area and to prepare maps of the distribution, geometry and lithology of the major geological units, which are presented in Chapter 3. Information on water levels, recharge and discharge, and on the hydraulic characteristics of the main units are given in Chapters 4, 5 and 6, respectively. A brief description of the model to be used for the drainage network design is given in Chapter 7.

Chapter 2

DATA COLLECTION PROGRAMME

2.1 DATA REQUIREMENTS AND BOUNDARY CONDITIONS

The type of information required by the model includes the following:

- thickness, elevation and extent of the major lithological units
- aquifer characteristics (hydraulic conductivity and storage coefficients)
- sources and amounts of recharge to and discharge from the area.

The accuracy of the model representation and the predictions made with the model will depend mainly on the reliability of the input data.

Some geological data has been obtained from existing reports prepared for various building projects carried out in West Bay over the past 10 years. These are given in Appendix 1. However, the reports do not contain sufficient information for the model requirements. Hence the main objective of the extensive data collection programme undertaken between April and November 1989 was to provide sufficient information of an adequate quality for the model.

The investigations were carried out within the boundaries shown in Figure 2.1. These boundaries were selected specifically to assist the modelling.

The eastern model boundary is defined by the coast and is represented as a "fixed head" boundary which thereby allows groundwater discharge to take place at the coast. The groundwater heads are fixed at sea level.

The western, northern and southern boundaries are located inland and have been chosen to coincide with particular groundwater contours so as to form "no-flow" boundaries. This eliminates the need to calculate groundwater inflow across these boundaries.

(a) Western boundary. This boundary coincides with the crest of a prominent NNW-SSE trending groundwater mound within the Dammam dolomite. The mound is centered beneath the Madina-Khalifa district and is caused by recharge from garden irrigation and leaking pipes (ASCO 1983). Groundwater flow occurs from the crest to the north-west and south-west, but not across the boundary. The position of this mound determines the western limit of the study region, and explains why this boundary has been selected a significant distance inland from the main area of concern.

(b) Northern and Southern boundaries. Both of these boundaries are parallel to the direction of groundwater movement, which is towards the coast. They therefore represent groundwater flow lines.

All of the model boundary conditions are thus well defined and represent the groundwater conditions in a realistic manner.

2.2 DRILLING AND TESTING PROGRAMME

2.2.1 Borehole Drilling Programme

A total of 58 boreholes were drilled within the model area during the present investigation. Locations are shown in Figure 2.1. Also given are the positions of building projects for which existing geological data are available. Prospect boreholes have been grouped into five series, prefixed GWS 1 to GWS 5. The depth of each series and their specific objectives are summarised in Table 2.1.

Table 2.1

Borehole no.	Depth	Water Table Elev.	Depth of Fill	Thickness of Dammam Limestone	Slug Tests	Pump Tests	Tidal Efficiency
GWS1/1-1/19	1 metre below water table	•					
GWS3/1-3/10	To top of Dammam Limestone						
GWS4/1-4/6	To base of Dammam Limestone						
GWS5/1-5/5	To top of Dammam Limestone						
GWS5/3 1/18 2/3							

Series GWS 1 and GWS 2 comprise 37 shallow boreholes drilled to a depth of 1m below the water table to obtain water level information.

The GWS 4 series comprise six boreholes drilled inland on the outcrop of the Dammam dolomite. Each fully penetrates the dolomite and has provided essential water level control data for this western part of the study area.

Series GWS 3 and GWS 5 comprise 15 boreholes drilled through the full thickness of the fill and coastal deposits to the top of the Dammam Formation. These were used to obtain information on the hydraulic characteristics of these deposits and for geological and water level data.

2.2.2 Testing Programme

Several techniques were employed to obtain information on the hydraulic characteristics:

input tests. These tests involve the instantaneous removal of a "slug" of water from the borehole and monitoring the subsequent recovery. They were undertaken on the series 3 and 5 boreholes to provide a low cost means of obtaining information on the hydraulic conductivity.

- pumping tests. These tests were carried out on the series 5 boreholes to provide information on transmissivity, hydraulic conductivity and storage coefficient.

- grain size data. The information available from grain size analyses given in earlier site investigation reports were used to estimate hydraulic conductivity

- tidal response. Values of storage coefficient were derived from a correlation of water level response to tidal fluctuations.

2.3 WATER BALANCE STUDY

The components of the water balance for the study area are discussed in Chapter 5. A parallel study is ongoing to quantify the recharge from garden irrigation and leaking pipes and the discharge of groundwater into sewers. This will cover the period from November 1988 to November 1989 and the results will be included in the Final Report.

Chapter 3

GEOLOGY

3.1 GENERAL

West Bay is an area of reclaimed land built out from an original, low lying coastline fringed with saline sand flats (Sabkhas). 'Hydraulic Fill', a mixture of sand, silt and gravel dredged from the adjacent sea floor, has been used for the reclamation. The fill, built up from the sea floor to form West Bay peninsula, is also spread some distance inland from the original coastline (Fig. 3.1). It overlies and largely conceals coastal silts and sands associated with the old shoreline. Some of this original material still crops out inland of the fill, but the precise position of the boundary between the two formations is uncertain (Fig. 3.1).

The bedrock of the area is the Upper Dammam Formation, a series of indurated fractured dolomites of Eocene age. Where the dolomite emerges from beneath the cover of fill and coastal deposits, along the line of the 4 m ground surface contour, there is a distinct break of slope. From this point the dolomite rises inland to over 20 m QND within the model area. A generalised map of the geology is shown in Figure 3.1 and a simplified cross section is presented in Figure 3.2.

From a hydrogeological point of view all formations in the area are permeable and can be considered as 'aquifers'. The impermeable base to the sequence is taken as the Midra Shale, a variably thick calcareous mudstone that underlies the Dammam dolomite. Above the shale there is free regional hydraulic connection between the fractured dolomite and the overlying coastal deposits and hydraulic fill. The water table passes laterally from the dolomite into the material above approximately along the line of the former coast (Fig. 3.2). Although there is free hydraulic connection on a regional scale, impersistent silt and mudstone horizons within the hydraulic fill and particularly the coastal deposits, provide local barriers to groundwater movement.

3.2 DISTRIBUTION AND LITHOLOGY

3.2.1 Dammam Formation

The entire study area is underlain by a series of fractured dolomites and limestones with a recorded thickness of between 13 and 35 m. Typically the formation is an off-white to grey dolomite, which in the upper 10 m contains numerous large vugs (cavities), mostly filled with carbonate mud. Commonly the vugs are between 2 and 20 cm in diameter and are not interconnected.

The upper 10 m has the appearance of a weathered horizon. Many of the open vugs have originated by the removal of soluble material infilling fossil shells (bivalves and gasteropods). The dolomite and carbonate mud mixture is generally indurated with much of the dolomite having a blue-grey siliceous appearance.

Small irregular fractures are common, although many have been re-cemented. Larger fractures are less common, but where present are frequently sub-horizontal, up to 30 cm in width and extend laterally for many tens of metres.

The lower part of the formation, below 10 m, is characteristically a zone of massive off-white dolomites with few vugs and little fracturing. This part of the sequence is significantly less permeable than the upper 'weathered' zone. (ASCO - 1983; JICA 1987).

Groundwater flow in the dolomite does not take place uniformly throughout the entire saturated thickness but along well-defined, widely spaced networks of fissures that have little or no hydraulic connection. Increased fracturing in the upper 10 m ensures that the highest permeability, and thus most flow, takes place in this part of the formation.

An impermeable base to the dolomite is formed by the Midra Shale, a variably thick sequence of brown and green carbonate shales alternating with thin dolomites. It is Lower Dammam in age and between 5-10 m thick.

The Midra Shale, acts regionally as a confining layer, preventing large scale vertical movement into and out of the overlying dolomite. The elevation of the top surface of the shale is shown in Figure 3.3. The main point of interest, apart from the general fall in elevation toward the coast, is the way in which this surface tends to mimic the ground topography. A casual comparison with topography (Figure 1.1) shows a tendency for high and low regions on the Midra surface to coincide with ground surface highs and lows.

The reason is straightforward. To a large extent, the topography is a product of subsidence following solution of evaporite deposits in formations lying below the Midra. This unsystematic collapse has led to the development of a large number of unconnected depressions that form a characteristic feature of the Qatar landscape. Because collapse is initiated by removal of material below the Midra, the shale itself subsides in the same way as the ground surface.

A result is that in places, the shale has been fractured and broken, allowing some limited vertical migration of groundwater. However, on a regional scale the amount of water transferred is very small so the shale can still be considered an efficient 'aquitard'.

The elevation of the top surface of the Dammam formation is shown in Figure 3.4. To the west of the 4 m ground contour the formation crops out at the surface, its top being reflected by ground topography. To the east, where it disappears beneath the cover of fill and coastal deposits, the dolomite dips gently toward the coast reaching its lowest point in the vicinity of the Sheraton Hotel, where it lies at - 7 m Q.N.D. Otherwise the most note worthy feature is a slight ridge which extends along the northern part of the peninsula.

3.2.2 Coastal deposits

The original coastline is characterised by a number of flat inter- and supra-tidal deposits known as 'sabkhas', interspersed with other areas of silts, sands and gravels. The sediments extend from below the former low tide mark to approximately the line of the 4 m ground surface contour.

Sabkhas are salt encrusted, flat lying areas of silt and sand. They are common in coastal areas of the Arabian Gulf (Evans *et al.*, 1969; Fooke et al 1985) Evaporation from shallow water tables within these flat-lying coastal areas provide an important mechanism for the discharge of coastward moving groundwater leading to the concentration of pore water and the precipitation of aragonite, calcite, gypsum, anhydrite and halite salts, all of which are commonly present in sabkha sequences.

In the West Bay area, sabkhas form the floor of several depressions enbaged along the original coastline. The major areas are shown in Figure 3.5. These have now largely been covered by a thin mantle of backfill. However, they still show through in small isolated patches, for example immediately to the south east of the West Bay sports stadium and alongside the coast road in the northern part of the region.

From a drainage point of view the areas of thinly-covered sabkha are important for two reasons. Firstly, they form depressions, where the water table is shallow and into which surface run-off concentrates and, secondly, they tend to have a low permeability. Together, these cause serious groundwater and surface water drainage problems.

The original coastal deposits form an extremely complex sequence of carbonate rich silts, sands and shelly gravels. Vertical and horizontal variation is such that it is impossible to devise any simple division. Correlation between boreholes is very difficult, even over distances of a few tens of metres. A good example is shown in Figure 3.6 which shows the lithology encountered in six boreholes drilled during construction of the West Bay Sports club. Although distances between boreholes are generally less than 100 m there is no consistent pattern in the sedimentary sequence.

At any single location the succession is likely to consist of an unpredictable sequence of silts, sands and shelly gravels with silty sands being perhaps the most commonly encountered lithology. The presence of silt horizons means that vertical movement of groundwater will be restricted locally. However, on a regional scale the succession can be treated as a single hydraulic unit due to the impersistent nature of individual beds.

The thickness of the costal deposits is shown in Figure 3.7. This ranges from zero, below parts of the West Bay peninsula, to over 6 m in areas inland of the former coastline.

3.2.3 Fill Deposits

The fill consists of two types of material:-

(a) 'Hydraulic Fill'. This is sediment dredged from the nearby sea floor and pumped as a slurry behind bunds to build up reclaimed areas. It comprises a melange of silty sands, shelly gravels and limestone cobbles. Because it is deposited as a slurry it is well mixed and thus tends to be more uniform in composition than the underlying coastal sediments.

(b) 'Desert Fill'. This material has been placed on top of the 'hydraulic fill' during later stages of the reclamation work. It consists of various types of natural and man made rubble bought from inland and dumped by lorry. It is much more variable in composition than the hydraulic fill. Pieces of wood, concrete and plastic are commonly encountered, along with large cobbles and boulders of dolomite. However 'desert fill' is patchily distributed and where present tends only to form the top metre of the succession.

As the hydraulic fill is simply re-worked coastal sediment, it is difficult to distinguish from the underlying coastal deposits. Consequently, the thickness and elevation of the base shown in Figures 3.8 and 3.9 are to some extent speculative. A means of distinguishing between the two has been to assume that all material to the east of the former shoreline lying above present day sea level is hydraulic fill.

The two maps are self explanatory. However, it is worth noting that the elevation of the base of the fill declines steadily eastward to below -3 m QND while the thickness increase uniformly in the same direction to over 8 m along the eastern coast of the peninsula.

Chapter 4

THE WATER TABLE

4.1 WATER TABLE ELEVATION

Each of the three major lithological units above the Midra Shale are in regional hydraulic continuity. Water is able to pass freely from one to the other despite the local presence of impersistent silt horizons in the coastal sediments. The water table is thus a composite of all three formations. It passes laterally from west to east through the dolomite, into the coastal deposits and finally into the hydraulic fill (Figure 3.2.).

The general configuration of the water table is shown in Figure 4.2. The main features to note are:

- to the north, south and west the model has "no flow" boundaries; the coast is a "fixed head" boundary set at sea level.

- there are steep gradients up to 1 in 150 in the western part of the area. This represents flow from the prominent groundwater mound marking the western boundary. The mound is built up to elevations in excess of 8m QND within the Dammam dolomites. It is caused by high recharge from garden irrigation and leaking water pipes within the Madina Khalifa district, compounded by local areas of low permeability in the dolomite.

- elevations within the sequence of fill and coastal deposits are less than 1m QND; on the peninsula they do not exceed 0.6m QND. Hydraulic gradients are low in the coastal area, reducing to as little as 1 : 4500 on the peninsula.

A distinct groundwater ridge extends along the southern part of the peninsula. This can be attributed to extensive irrigation along the central reservations and sides of the roads converging on the Sheraton Hotel at the head of the peninsula. Apart from this ridge there is little other evidence that, to date, the water table has been affected by mans activities. However, this probably reflects the current lack of development in the backfilled areas of West Bay.

When large scale housing projects are established in the peninsula and in areas along the former coastline, it is likely that water levels will begin to rise significantly. The ridge along the southern side of the peninsula illustrates how water levels can rise in response to urban development.

4.2 DEPTH TO WATER TABLE

The depth to water is shown in Figure 4.2. Two distinct provinces exist; the backfill and coastal deposit areas, where groundwater generally lies within 3 m of the surface, and the dolomite outcrop where depths are greater than 3m.

The dividing line between the two occurs approximately at the 4m ground surface contour.

The dolomite outcrop area faces no immediate threat from rising groundwater levels since depths are commonly in excess of 10m. However, the backfill-coastal deposit areas are obviously at risk.

The areas with the shallowest water table are not, as might be expected, nearest the coast, but in a belt which extends parallel to and just inland of the former shoreline (Figure 4.2). The reason is quite straightforward; the area has the lowest ground elevation in the region.

Backfilling of the peninsula has raised ground elevations in this area to over 3m QND in many places. Around the northern and eastern coasts it is above 5m QND. However, although backfill has been extended westward and inland of the original shoreline, it has not been built up as high as the peninsula itself. The result is that the old coast has been left at a slightly lower elevation than areas further east.

It is this belt of low lying ground that was most severely affected by the heavy rainstorms in February 1988. During this month the total rainfall was 140mm, with 39.8mm falling within a 24 hour period. A particularly badly affected area was in the vicinity of the West Bay sports stadium. Here the combination of a topographic depression, shallow water table and the presence at the surface of patches of silty sabkha combined to give rise to conditions likely to result in flooding.

This particular flood event was caused by an exceptionally intense rainstorm and much of the problem lay with inadequate surface drainage from low lying areas. Such events, fortunately, are rather rare and the effects could largely be countered by the installation of surface drains. There is, however, a longer term and potentially more serious problem facing the area. This is the threat posed by increased recharge resulting from future development in the region.

Increased recharge, will raise groundwater levels very close or even to the surface. When this happens the problem is long term and can only be alleviated either by reducing recharge or more practically by installing a network of groundwater drains.

It is the area having water depths of less than 1m that most urgently require the installation of such drains. The problem here is compounded by the presence of poorly permeable sabkha deposits at or very near the surface. Ultimately, however, the whole of the West Bay peninsula, with the possible exception of the raised outer rim, will require groundwater drains.

4.3 WATER TABLE FLUCTUATIONS

Water levels in West bay are strongly influenced by tidal fluctuations. The relationship between groundwater levels and tides has been examined for 3 sites which were fitted with water level recorders. Boreholes 1/18, 2/3 and 5/3 are positioned at 150m, 500m and 850m from the coast respectively (Figure 2.1).

Figure 4.3 is a plot of the water level response against tidal fluctuation for the 3 sites. The very limited response of the borehole closest to the coast, 1/18, is attributed to clogging of the borehole, which only penetrates 0.5m below the water table. The data from this site have, therefore, been discounted. The two other boreholes show a contrasting response in terms of both the amplitude of fluctuations and the time lag before a response is recorded.

Borehole 2/3, which is closer to the coast, shows the least lag. Water levels responded to tidal changes within 1 hour compared to 6 hours for 5/3.

The amplitude of water level fluctuation of the two sites is also very different. At 2/3 the response is very pronounced with daily fluctuations up to 40cm. In contrast the response at 5/3 is measured in millimeters, so small that it is barely perceptible on the scale used in Figure 4.3.

The water level response can be expressed in terms of "tidal efficiency", which is the ratio of water level rise to a given tidal change expressed as a percentage. For site 2/3 the efficiency is 32% compared with only 2% for site 5/3. These data can be used to derive an estimate of aquifer storage since amplitude, lag and tidal range are linked to storage. This aspect is discussed further in Chapter 6.

More work is planned concerning tidal fluctuation as a means of deriving aquifer storage values. The water level recorders will be moved to a number of other sites to build up a better picture of how water levels respond to tides in different parts of the area.

Chapter 5

RECHARGE AND DISCHARGE

Rising groundwater levels result from an excess of recharge over discharge. The design of a drainage network to control water levels in the West Bay area will require the identification and quantification of the mechanisms controlling recharge to and discharge from the aquifer system.

The sources of recharge and disharge identified in West Bay are as follows:

Recharge:

- rainfall
- lateral inflow
- leaking water pipes
- garden irrigation

Discharge:

- evaporation
- lateral outflow
- downward leakage
- leakage to sewers.

These are described in the following sections.

5.1 RECHARGE

5.1.1 Rainfall

Rainfall in Qatar is low and characterised by extreme variability in both space and time. Recharge from this source is likewise very variable and difficult to predict from year to year and from place to place. Records from the airport (1962-1988) in Doha serve to illustrate the main characteristics of the local rainfall distribution. (Figures 5.1 and 5.2). A shorter record also exists for the port (1979-1988).

The annual average rainfall at the airport, the station with the longest record, is 74.7mm; at the port it is 66.2mm. Most of this rainfall occurs between November and May. During the 26 year period of record at the airport the maximum and minimum annual totals were 302.8mm (1964) and 0.4mm (1962) respectively. Such extreme variability from year to year makes the concept of "mean annual" rainfall rather meaningless.

Rainfall up to in Qatar usually occurs as intense localised storms. Individual storms can account for 65% of the annual total. Table 5.1 shows the probability of annual and storm rainfall totals for Doha airport.

Table 5.1 Probability of annual and storm rainfalls: Doha airport

Recurrence interval (years)	Annual (mm)	Storm (mm)
5	35	7
10	75	18
15	120	40
20	180	62
25	260	105
30	360	170

Not only do the intensities of storms differ from year to year but for any one event the distribution of rainfall over small areas can be very different as the storm cells are very localised and sweep over relatively narrow tracts of land.

All of these factors make it extremely difficult to predict the recharge from year to year. There are also other factors that will control the level of recharge from this source. These include:

- intensity of the storm
- permeability of the surface material
- pre-existing soil moisture conditions
- rate of evaporation
- topography (eg. depressions allow concentration of run-off)

Given the multitude of factors influencing rainfall recharge the only practical way of quantifying the process is to relate changes of water level with individual rainstorms. In this way it is possible to calculate recharge for any particular intensity and duration by simulating the event with the groundwater model. This is why it is necessary to have rainfall and water level records for a complete rainy season. Such data are currently being collected and will be discussed in the Final Report in April.

5.1.2 Lateral Flow of Groundwater from Inland

A major source of recharge is the lateral movement of groundwater flowing from the groundwater mound developed along the western boundary. The model will be used to quantify this inflow using the Darcy equation:

$$Q = T^{*}I^{*}W$$

where: Q = Flow
I = Water table gradient
W = Width of aquifer over which calculation is taking place
T = Transmissivity of the aquifer

5.1.3 Lateral Flow of Seawater at High Tide

During high tides sea level is raised significantly above groundwater levels near to the coast for short periods. When this happens seawater will flow inland until the tide recedes.

This effect will be built into the model by setting the initial tidal conditions at high tide. Flow can then be calculated as described in section 5.1.2.

5.1.4 Leaking water pipes

It has been shown by ASCO (1983) that water distribution system losses in Doha amount to over 20%. This figure is comparable to distribution systems throughout the developed world.

We anticipate a similar scale of loss for West Bay. The volume will increase as the urban area develops. Leaking water pipes will constitute a significant source of recharge to the area and contribute to a steady long term rise in water level.

In order to establish the scale of leakage a water balance study is currently underway to calculate losses for the year November 1988 to November 1989. The results of this work will be incorporated into the model for calibration purposes. Once calibrated the effects of increased losses in the future can be calculated.

5.1.5 Garden watering

Another significant man-made source of recharge is garden irrigation. Again this will tend to increase as development of the area proceeds. Estimates of contributions from this source are to be made within the water-balance survey presently being carried out.

5.2 DISCHARGE

5.2.1 Evaporation

Under natural conditions significant groundwater discharge occurred by direct evaporation from the shallow water table underlying the areas of coastal sabkha.

Discharge from coastal sabkhas is fairly constant throughout the year (Pike, 1971). In eastern Saudik Arabia, rates ranged from 1.1mm per day in the winter to 1.8mm per day in the summer.

In West Bay many of the original sabkha areas have now been covered with up to 1m of backfill. Nevertheless, it is expected that where water levels are within 1.5m of the surface a significant amount of direct evaporation will take place. This discharge will be built into the model using published data to estimate the amounts involved.

5.2.2 Lateral Groundwater Flow to the Coast

This element of discharge will be calculated by the model in the same way as for flow into the area (see 5.1.2).

5.2.3 Downward Leakage to Lower Aquifers

As water levels in the formations above the Midra Shale are built up due to increased recharge, some leakage into underlying aquifers through fractures in the Midra might take place. However, the permeability of the underlying formations is sufficiently low (ASCO 1983) to suggest that the quantities involved will be small and can be ignored.

5.2.4 Leakage into Sewerage Pipes

Leakage of groundwater into unpressurised sewerage pipes set below the water table will be a major source of discharge. The sewers will in effect be acting as groundwater drains. This aspect of the work is being incorporated into the water balance study.

Chapter 6

AQUIFER CHARACTERISTICS

6.1 GENERAL

Values of hydraulic conductivity (K) and storage coefficient (S) are required for the model. Transmissivity (T) is computed by the model from the water level and aquifer base information.

Preliminary estimates of K have been derived from pumping and input tests carried out as part of the field programme and supplemented by grain size data available from previous site investigations in the general area. Values of S were derived from the pumping tests and by comparing water level changes to tidal fluctuations.

6.2 HYDRAULIC CONDUCTIVITY

6.2.1 Estimates from Pumping Tests

As no pumping test information was available for the study area, a programme of pumping tests was undertaken at sites GWS/5/1 to GWS/5/5 to provide information on the hydraulic characteristics of the unconsolidated sequence.

The locations of the test sites are shown in Figure 1.2. A test borehole and two observation boreholes were drilled at each site during July and August 1989. Each borehole fully penetrates the aquifer sequence and is screened from the water table to the base of the aquifer. General information on each site is as follows:

Table 6.1 Details of pumping test boreholes

Site Number	Ground Level Elevation (m)	Depth to Water (m)	Depth to Bedrock (m)	Saturated Thickness (m)
5/1	2.95	2.80	6.5	3.7
5/2	2.08	1.26	4.0	2.7
5/3	2.43	2.20	4.5	2.3
5/4	2.30	2.40	5.0	2.6
5/5	2.59	1.60	4.0	2.4

In all cases the sequence tested consists of silty fine to coarse sand with gravel. The log descriptions suggest that the fill and coastal sands are of a similar lithology at each site, but, whilst the coastal sands form the whole saturated sequence at sites 5/1, 5/2 and 5/3, these form only 65% and 73% of the saturated sequence at sites 5/3 and 5/4.

The pumping tests were carried out during October and November 1989. Transducers were used to measure water levels in the test borehole and each observation borehole during the tests. Test rates ranged from 0.5 to 5.75 l/s.

A pumping test could not be carried out at site 5/1 due to low yield. Only a single constant rate test was carried out at 5/5. Three tests of up toabout 2 days duration with intervening recovery, were performed at each of the sites 5/2, 5/3 and 5/4. A summary of the test programme is given in Table 6.2.

Table 6.2 Summary of pumping test programme

Borehole Site	Date of Test	Rest Water Level (m) below datum	Rate l/s(m^3/d)	Duration (mins)	Recovery (mins)
5-2-1	25/10/89 (A)	1.280	5.67 (490)	77.5	50.5
5-2-1	29/1089 (B)	1.260	5.75 (497)	450.0	45.o
5-2-1	30/10 → 1/11/89 (C)	1.245	5.75 (497)	2885.0	120.5
5-3-1	5/10/89 (A)	1.990	0.71 (61)	61.0	2.0
5-3-1	7/10/89 (B)	2.00	0.967 (84)	345.0	40.0
5-3-1	8/10 → 9/10/89 (C)	2.010	0.85 (73)	*1320.0	152.5
5-4-1	15/10/89 (A)	2.830	0.85 (73)	255.0	
			0.967 (84)	44.5	
5-4-1	16/10/89 (B)	2.550	1.060 (92)	240.0	180.0
5-4-1	17/10 → 19/10/89 (C)	2.490	0.95 (82)	3225.0	120.0
5-5	5/11/89 (A)	1.639	0.50 (43)	1605.5	60.0

* - Full data still to be tabulated

The drawdown data are affected by tidal fluctuations. As yet a satisfactory correction for these fluctuations has not yet been obtained. This will require further information on the natural water level response to tides. Other factors influencing the analysis of the pumping test data include the following:

- the aquifer is thin and the test well data are affected by a reduction in aquifer thickness of 90 and 96% at 5/3 and 5/5 and by 52 and 54% at 5/2 and 5/4

- well losses influence the test well data

- well storage may influence the early data from the test well at the low rates of pumping

- gravity drainage appears to be affecting the data, but because of the tidal effects it is not yet possible to derive specific yield estimates from the late data

- drawdowns at the observation wells are small and are therefore particularly affected by the tidal fluctuations

- the sequence is very variable and boundary effects or lateral variations in transmissivity may be affecting the data.

An initial estimate of T was made using Logan's approximation method based on specific capacity prior to the onset of gravity drainage, which generally occurs after about 5 minutes. This method tends to underestimate T if well losses are significant. The results are included in Table 6.3. The time-drawdown data for each borehole appear to fit a Boulton or Neuman type curve which indicates water table conditions. However, since the type curve methods are likely to produce unreliable and ambiguous results unless the data can be corrected for the tidal fluctuations, these analytical methods have not yet been applied.

In order to obtain preliminary estimates of T and S, semi-log methods have been used. These were applied to the early time-drawdown data, distance-drawdown data and to the recovery data, although it should be recognised that the application of these methods are subject to various constraints. Nonetheless, the values derived are in reasonable agreement with the approximation method and with the input tests at each site.

A summary of the preliminary results from the pumping tests is given in Table 6.3.

6.2.2 Input tests

Input tests were also used to provide estimates of K. The values derived are representative only of the water bearing material close to the well and are therefore usually less reliable than pumping tests.

These were performed on fifteen wells: ten in the series 3 boreholes (diameter 168mm) and five in the series 5 boreholes (diameter 203mm). The locations are shown in Fig 12.

Each test involved the rapid removal of a known volume of water and monitoring the subsequent water level recovery with a pressure transducer connected to a millivolt chart recorder until equilibrium conditions were re-established.

The results obtained from the tests were analysed using methods developed by Cooper (1967) and by Bouwer & Rice (1976). Both methods assume that well losses are negligible and that the aquifer is homogeneous and isotropic.

The Cooper method takes into account both well storage and aquifer storativity, whereas the Bouwer & Rice method takes into account well storage but not aquifer storativity. The Cooper method involves a semi-log plot of

Table 6.3 Preliminary estimates of aquifer characteristics from pumping test analyses.

			5/2	5/3	5/4	5/5
1.	Logan Approximation @ t = 5 mins.					
		Tm^2/d	1240	90	145	140
		Km/d	460	48	69	72
2.	Distance-drawdawn (early data)	T		125	105	1975** ?
		K		40	45	820 ?
		S		0.2%	0.2%	$3x10^{-4}$
3.	Time-drawdown (Jacob)	Test well				
		Te	1300	25	80	140
		Tl	80	?		5
		K	30 - 480			
		Observation 1				
		T	6000 ?**	105	70 - 90	75
		S	11%	0.3%	0.25%	0.2%
		Observation 2				
		T	14000 ?**		115 - 135	
			11%		0.15 - 0.2%	
4.	Recovery			100	140	

Te = early data
Tl = lata data

* = rapid
** = influenced by leakage

values of head (H) divided by initial head (Ho) against log time (t). The resulting curves are then matched with type curves to obtain a value of T from the following calculation:

T= r/t (where r is the radius of the well)

Values of K are then obtained from K=T/D, where D is the saturated thickness.

The Bouwer & Rice method involves the plotting of head change against t on a semi-log scale. A dimensionless parameter, C, was determined from a curve relating C to L/r, where L is the length of the perforated section of the screen. The various values were then substituted into the following equations to obtain K:

$$\ln R_e/r_w = \left[\frac{1.1}{\ln(H/r_w)} + \frac{C}{L/r_w} \right]^{-1} \tag{1}$$

$$K = r_c^2 \frac{\ln (R_e/r_w)}{2L} \frac{1}{t} \ln \frac{Ho}{H} \tag{2}$$

where : C = dimensionless parameter
L = length of the perforated screen
R_e = effective radius over which head changes
r_c = radius of the well casing
r_w = well radius

Table 6.4 gives the results obtained by these methods of analysis. In two of

Table 6.4 Results of Input Tests

Borehole	Nos. of tests conducted	Saturated thickness (m)	Zones of saturation	K(m/d) Method of analysis	
				Cooper et al	Bouwer & Rice
3/1	1	3.20	limestone	too high for test	too high for test
3/2	3	dry	limestone	-	-
3/3	1	3.28	??	too low for test	too low for test
3/4	5	3.24	sand	242	20 - 35
3/5	5	1.57	fill	18	15 - 81
3/6	4	4.70	sand	338	141 - 398
3/7	4	2.05	??	No type fit	33
3/8	4	2.05	sand	135	55 - 228
3/9	4	3.70	fill/sand	12	35
3/10	4	2.50	fill/sand	55	11 - 34
5/1	1	3.64	??	too low for test	too low for test
5/2	5	3.90	??	No type fit	40
5/3	4	2.39	??	No type fit	9 - 130
5/4	3	2.60	sand	196	28 - 88
5/5	5	2.40	sand	322	22 - 103

Nos. of boreholes tested: 15
Borehole diamter - 3 series: 168 mm
5 series: 203 mm

the boreholes, 3/3 and 5/1, the tests conducted were unsuccessful due to low permeability. The test at site 3/1 was unsuccessful due to exceptionally high permeability, probably associated with a fracture zone. At three sites, 3/7, 5/2 and 5/3, none of the input tests produced curves which matched any of the Cooper type curve. In these cases a solution was obtained from the Bouwer & Rice method.

A wide range of K values was obtained from the input tests at the different boreholes. This indicates the hydraulic conductivity of the sequence to be very variable, although it should be recognised that this variation may in part be due to the subjectivity of the type curve methods themselves. Nonetheless, the preliminary results suggest that K ranges from about 15 to 80 m/d for the fill deposits and from about 10 to 400 m/d for the coastal deposits, which are consistent with those obtained from the pumping tests and from grain size analysis.

6.2.3 Grain size estimates

Various methods are available to derive estimates of K from grain size analysis data, although estimates based on this approach are usually applied only if other more reliable and representative estimates are unavailable. If such data can be calibrated, for example from pumping tests, they can provide a low cost means of providing information on the spatial variability of K but are more accurate where the sequence is isotropic and uncemented and the samples are representative.

Although grain size analyses were not undertaken on the samples collected during the field programme, such data are available for the general area from site investigations. The distribution of sites with grain size data is shown in Figure 1.2.

The specific surface method was used to provide initial estimates of K from the grain size data. A value of 50000 was assumed for the constant used for this method, but until this value can be confirmed by calibration with the pumping test and input test information, the K values should be considered as indicating only the relative distribution of K for the main parts of the unconsolidated sequence.

The samples for which grain size data are available were assigned to either the fill or coastal deposits based on an interpretation of the borehole log. The number of such samples relating to each were 68 and 38, respectively.

A plot of the K values (Figure 6.1) shows a skewed distribution. The values were therefore converted to a logarithmic form to produce a lognormal distribution (Figure 6.2). Table 6.5 gives the ranges and arithmetic and geometric means in m/d. The geometric mean is about 50% of the arithmetic mean and should be a more accurate estimate of the mean K as the few high K values have less effect on the geometric mean. The results indicate that the K of the fill and coastal deposits are very similar, although there is a wide variation in the K of both types of deposits. The constant applied was based on well sorted, marine sands and, despite the variability in K, produces a mean K similar to that derived from the input and pumping tests.

Table 6.5 Grain size permeability, values.

	Fill	Coastal Deposits
Min K	2.13	2.31
Max K	515	277
Arithmetic mean K	68.6	53.6
Standard Deviation	89.4	64.3
Geometric mean	35.4	26.5

A frequency analysis was also undertaken by assigning K values to classes in units of 10 m/d. These were then expressed as a cumulative percentage to overcome the difference in the number of samples from each type of deposit. The results are plotted as Figure 6.3, which further confirms the similarity in K of the two types of deposit.

6.3 STORAGE COEFFICIENT

The pumping test drawdown data still require tidal correction to the drawdown data. Consequently, it is not yet possible to obtain estimates of specific yield from the test data. The early drawdown data indicates a storage coefficient of 0.2% (Table 6.3), although this must be regarded as a preliminary estimate. The effective specific yield is likely to be higher, perhaps in the range of 10 to 30% depending on the silt and gravel content.

As an alternative approach a correlation between the tidal and water level fluctuation has been attempted using a method developed by Ferris, 1951. There are only three sites (GWS 1/18, 2/3 and 5/5/3) with sufficient information for this approach and realistic values could only be obtained for site GWS 2/3 ranging from 7 to 14% for T values of 50 to 100 m2/d.

Nonetheless, this method could provide a more reliable estimate of specific yield than that from the pumping tests and it is recommended that further data is obtained at other sites at different distances from the coast.

Chapter 7

MODEL DESIGN

7.1 INTRODUCTION

The aims of the numerical groundwater modelling are firstly to predict likely areas of flooding resulting from rainfall, leakage and irrigation recharge and secondly to suggest a possible drainage system designed to alleviate such flooding problems.

A two-dimensional areal representation of the study area would not adequately fulfil these objectives since the water table is present in all of the three main types of material (fill, coastal deposits and Dammam dolomite). In addition, saturated and unsaturated zone conditions need to be included, as well as flexibility in representing the study area by mesh discretisation.

However, only a few three-dimensional models can adequately represent the particular aquifer conditions in West Bay, due to their complexity and computational restrictions with respect to computer core storage and central processor unit time requirements. The model selected for this study is FLAMINCO (Flow And Migration In Non-conservative Contaminants), which is both suitable and efficient. Only the flow component of this model is considered in the present work. This model is described briefly below.

7.2 MODEL DESCRIPTION

Flaminco is three-dimensional finite element model which includes both saturated and unsaturated zones. Flow is governed by Darcy's law. Nonlinearities due to unsaturation, infiltration, evaporation and seepage faces are all considered. The model is cost-effective, utilising a slice successive over-relaxation matric solution scheme, with Picard iteration for treating nonlinearities and element integration using influence coefficient techniques.

7.3 MODEL DEVELOPMENT

7.3.1 Data sources

Data sources, such as those relating to aquifer geometry and properties, recharge and discharge, are described in previous sections of this report. Initial head conditions are derived from water levels at boreholes, corrected for the effect of tidal cycles. Data requirements for the model are specified either at nodal locations or for elements by interpolation from the point data.

7.3.2 Study Area and Boundary Conditions

The eastern model boundary is located at the present coastline (Fig. 1.2). Time-variable fixed head boundary conditions are used to represent changes in the tide along this boundary. Only high tides are considered, being believed to represent a 'worst-case' boundary condition for flooding inland.

The northern, north-western, south-western and southern boundaries are located inland along no flow boundaries suggested by water level contour maps. The south-western boundary is located on the long axis of a recharge mound. The northern, north-western and southern boundaries lie parallel to the direction of groundwater flow.

In a vertical direction, hydraulic fill, natural coastal deposits and Dammam dolomite are considered, the base of the aquifer being represented by the top of the underlying Midra shale, which is assumed to be impermeable.

7.3.3 Model Discretisation

The aquifer is represented by 161 nodes in each of the 9 nodal layers in the vertical direction giving a total of 1449 nodes, and by 280 three-dimensional triangular prism elements in each of the 8 corresponding layers of elements, giving a total of 2240 elements. The top layer grid is shown in Figure 7.1. Three layers of elements are used to represent the hydraulic fill, three layers for the natural coastal deposits and two layers for the Damman limestone.

In order to best consider gradients in material properties, such as hydraulic conductivities, between the three distinct material types, model layering was introduced just above and just below horizons between the three distinct material layers. Consequently, the 9 nodal layers are located as follows:

- (i) at the surface,
- (ii) at 0.1 m below the surface,
- (iii) at 0.1 m above the base of the hydraulic fill,
- (iv) at the base of the hydraulic fill,
- (v) at 0.1 m below the base of the hydraulic fill,
- (vi) at 0.1 m above the base of the natural coastal deposits,
- (vii) at the base of the natural coastal deposits,
- (viii) at 0.1 m below the base of the natural coastal deposits, and
- (ix) at the base of the Dammam limestone.

Only the Dammam dolomite is present in the vertical at all locations in the model area, as the natural coastal deposits and hydraulic fill are more limited in areal extent (for example, see cross-sectional diagram of Figure 3.2). As such, some of the elements suggested above are redundant. For example, towards the south-western boundary only two layers of elements (together representing the Dammam limestone) exist within any vertical, whereas towards the coast a full eight layers of elements (representing all three distinct material types) exist within a vertical.

The model has been designed so that redundant elements can be assigned a null material property. In effect this means that their respective element matrices are not evaluated and are not assembled into the global matrix equations. Some 23% of the 2240 three-dimensional elements are eliminated in this way, leaving a total of 1712 elements to be evaluated.

Figure 7.1 shows the configuration of the finite element mesh design. This was based on a number of criteria :

(i) model boundaries were selected as indicated above,

(ii) sides of elements are located along main roads,

(iii) nodes are located at roundabouts and intersections of main roads,

(iv) the coarsest mesh is used for areas of Dammam outcrop, except towards the south-western boundary where steep groundwater gradients required some fining of the mesh,

(v) a fine mesh is used for areas of hydraulic fill outcrop,

(vi) the finest mesh is used for areas of natural coastal deposit outcrop,

(vii) sides of elements are located along minor roads in areas of natural coastal deposits,

(viii) each surface element represents land that is predominantly developed, or to be developed or is not planned to be developed,

(ix) no surface element has all three surface nodes assigned fixed heads,

(x) since most drains were expected to be located alongside roads in areas of natural coastal deposit outcrops and drains were expected to be assigned fixed heads, not more than one side of any element in this region should represent a road in order to prevent overconstraining the model solution.

7.4 ANTICIPATED MODEL SIMULATIONS.

It is anticipated that four stages of numerical simulation will be performed. These are as follows:

- calibration of the model against field data
- simulation of a range of storms and prediction of areas liable to flooding
- simulation of changes in leakage and irrigation due to increased development and prediction of areas of floodings
- representation of a drainage system designed to alleviate flooding problems.

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Topography (m)

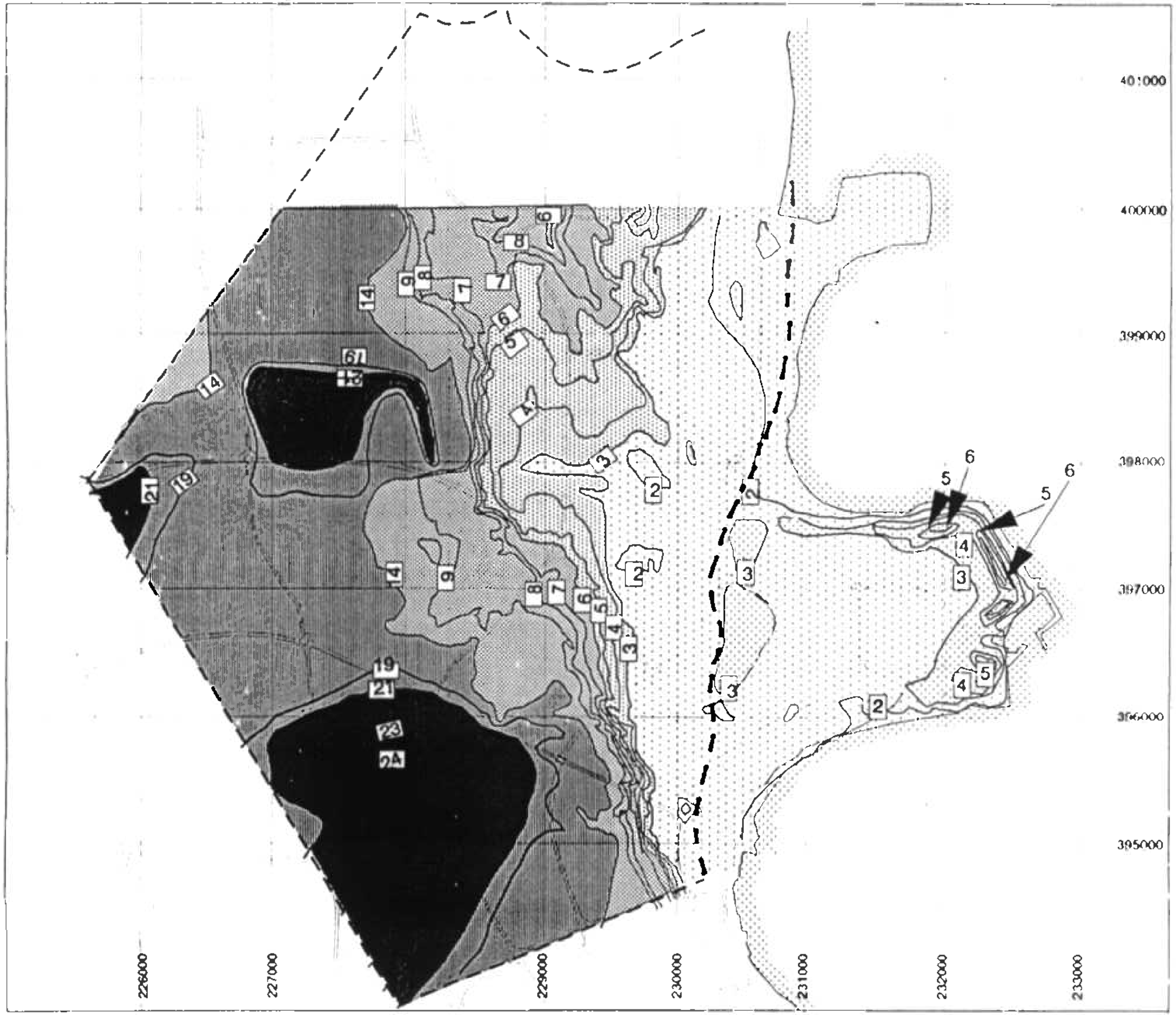


Figure 1.1

Areas of Development

Figure 1.2

Location of Project boreholes

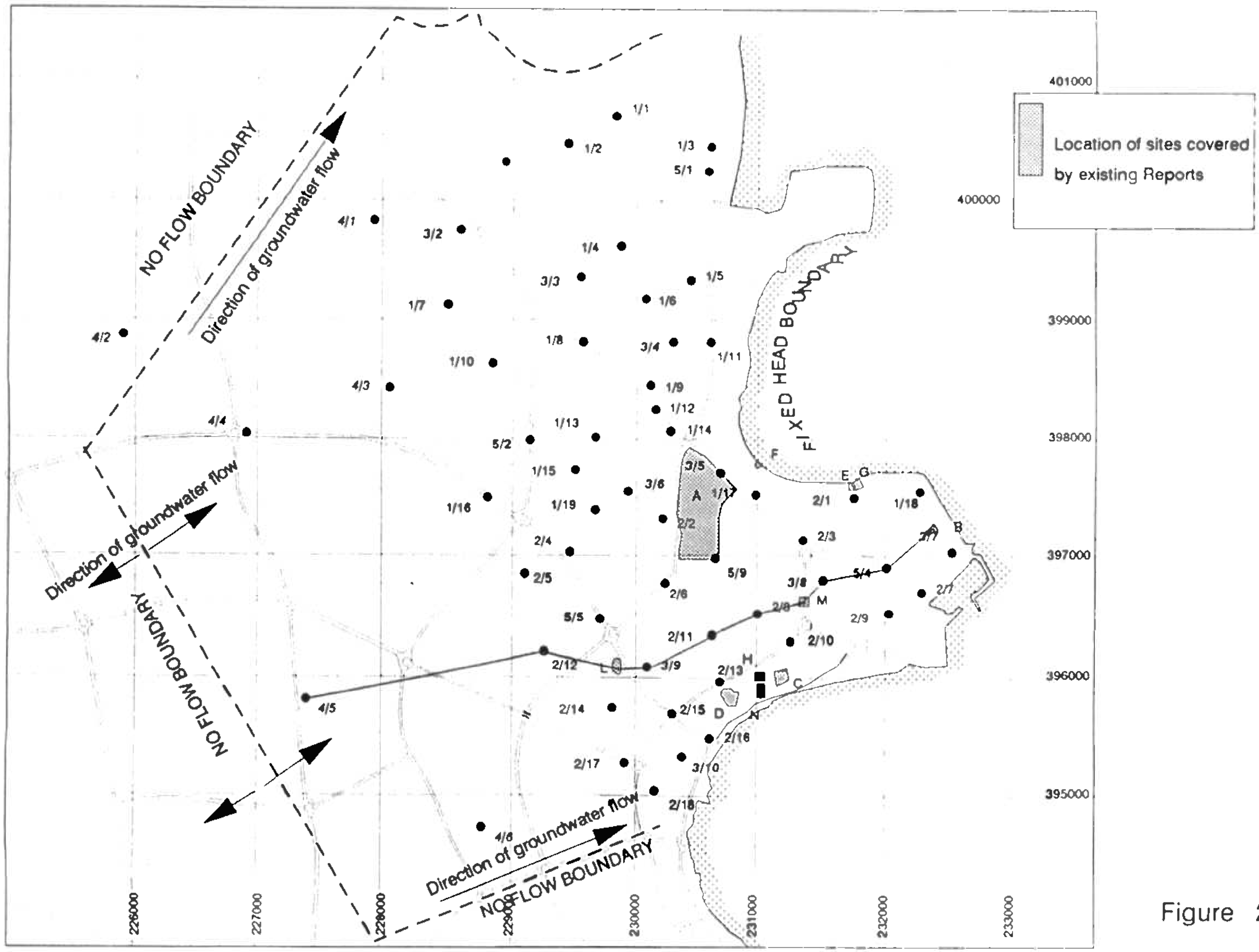


Figure 2.1

Outcrop Geology

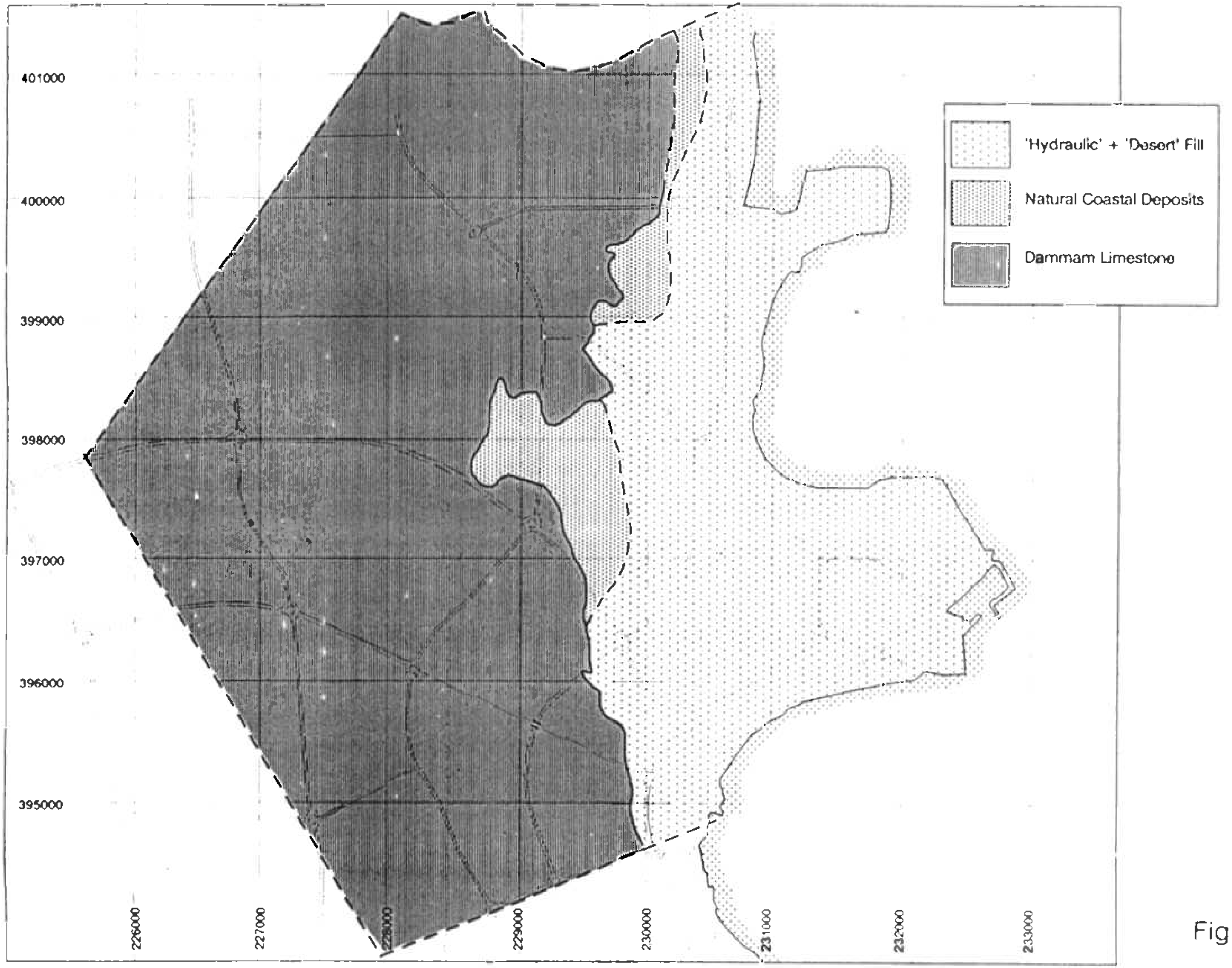


Figure 3.1

Geological Cross Section (See Figure 2.1)

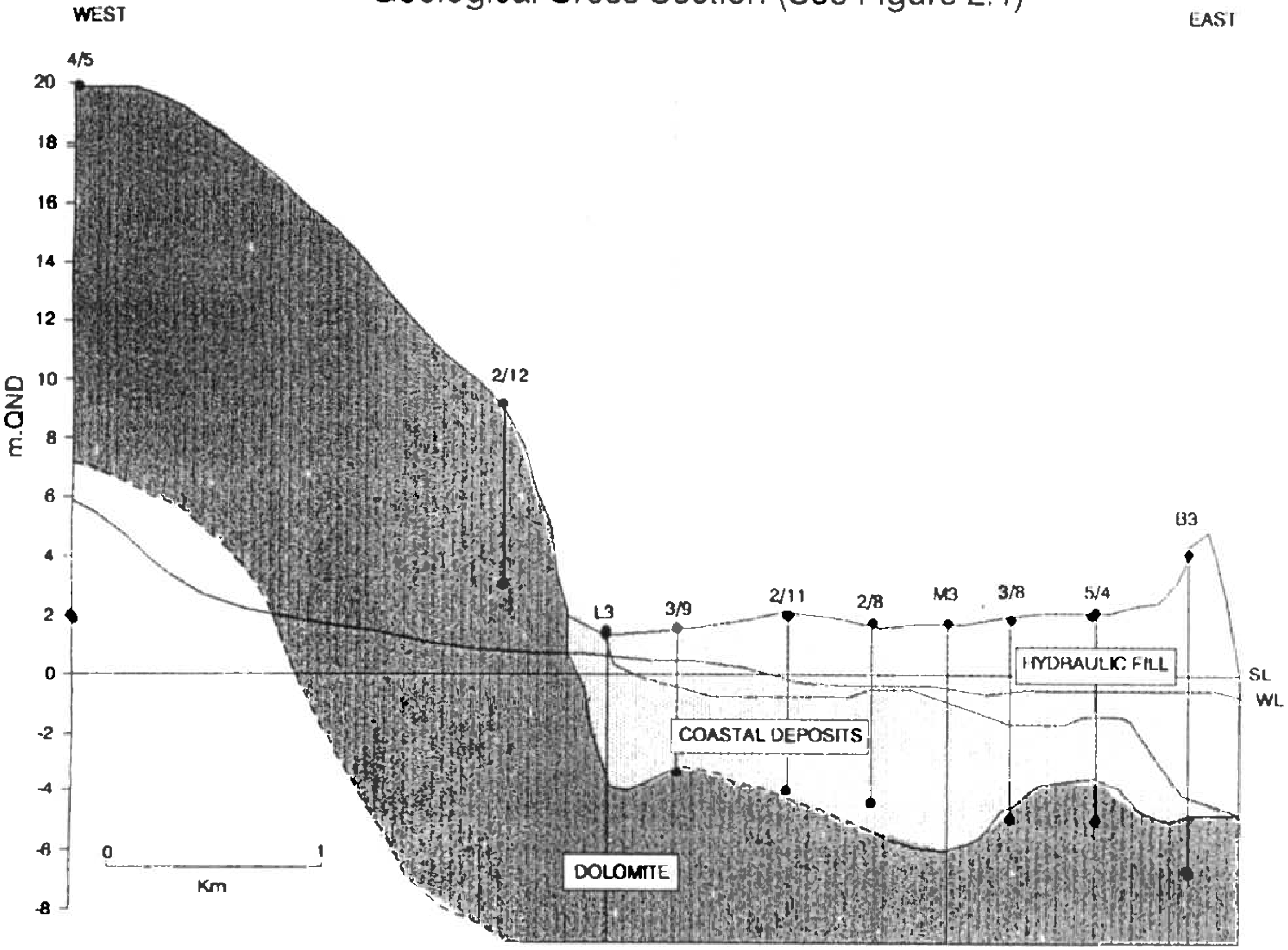


Figure 3.2

Elevation of top surface of Midra shale (m)QND

Figure 3.3

Elevation of top surface of Damman Formation (m)QND

Figure 3.4

Areas of Sabkha along original coastline

Figure 3.5

Cross section showing variability of coastal deposits

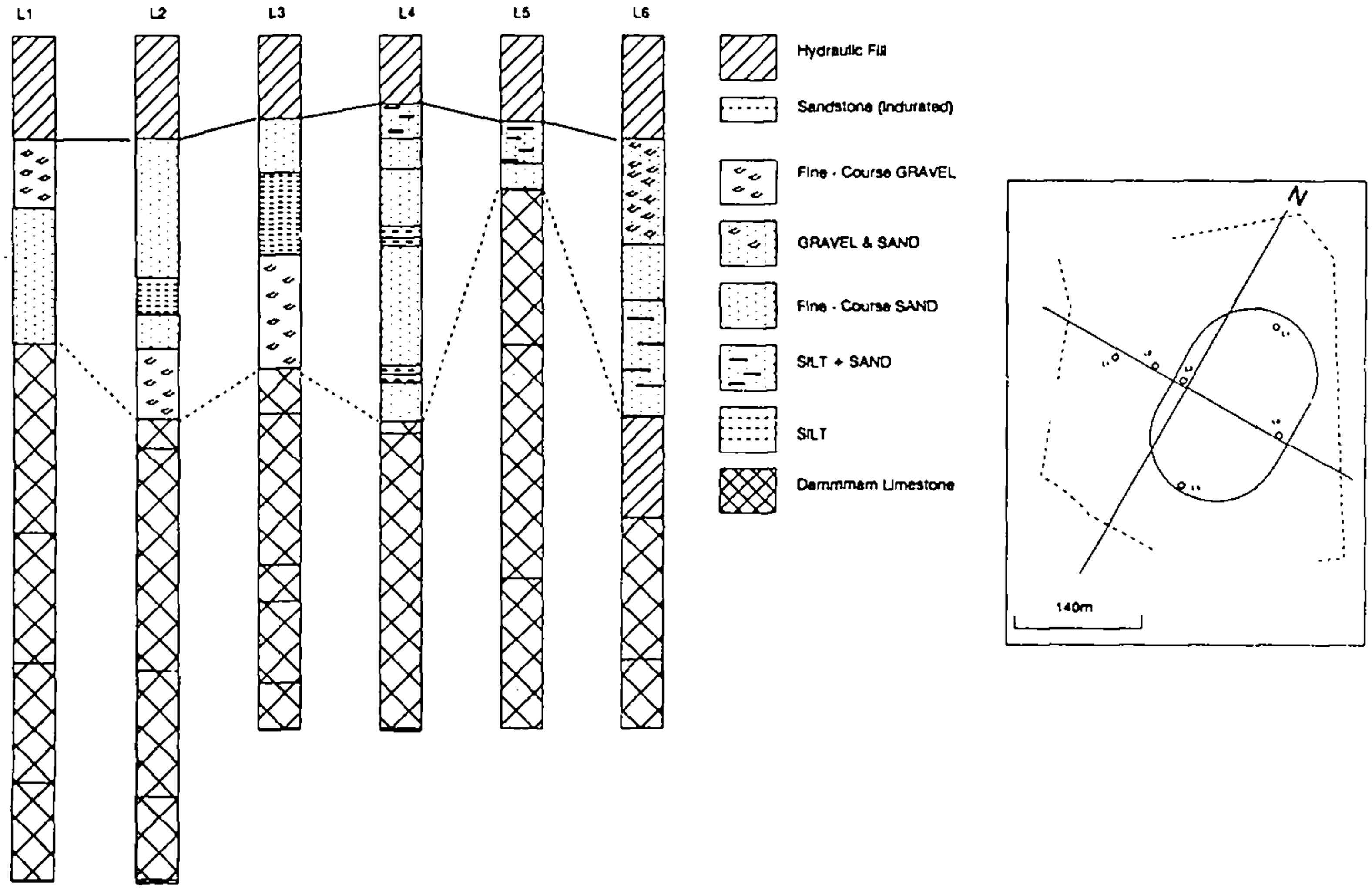


Figure 3.6

Thickness of Coastal Deposits (m)

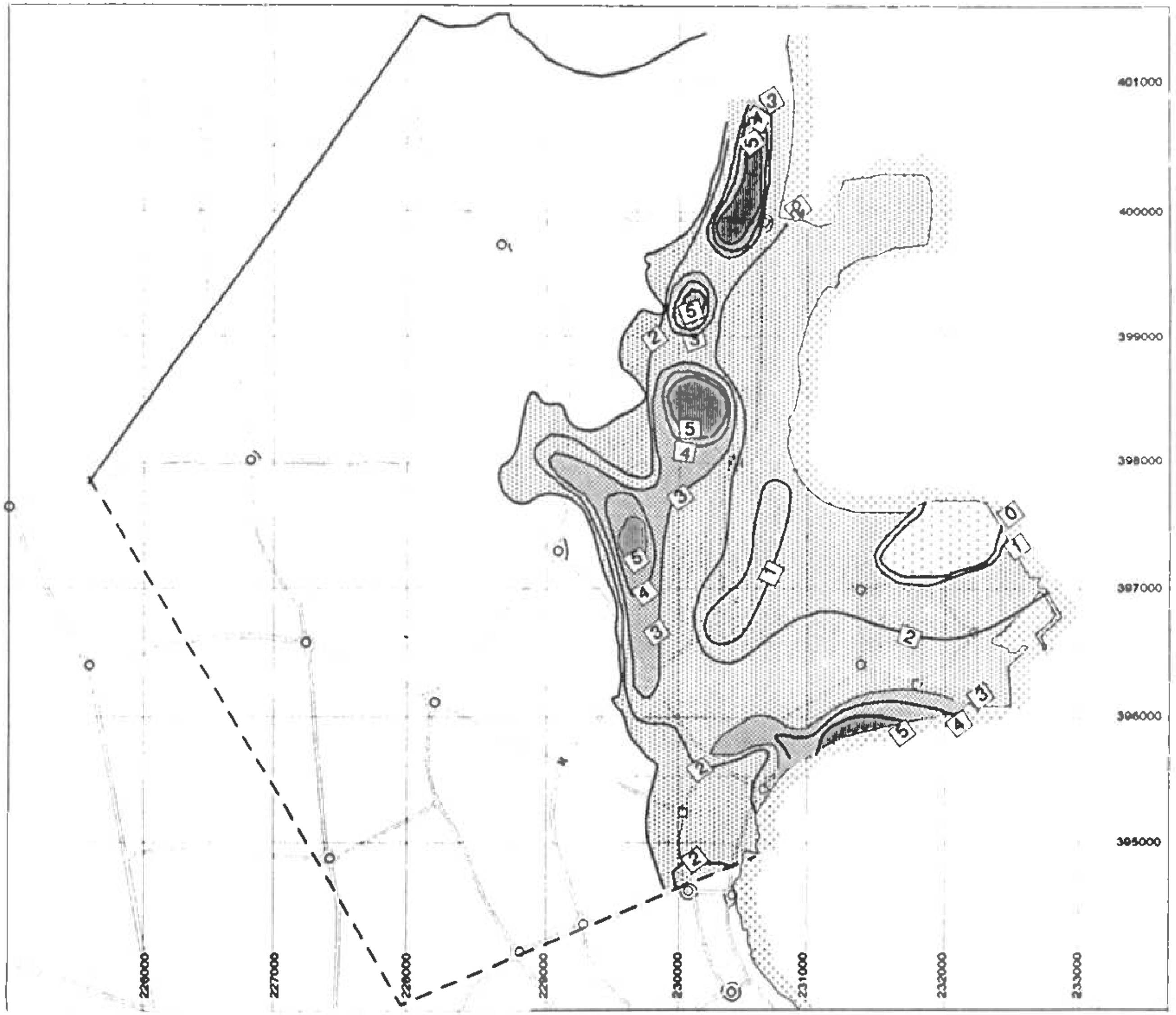


Figure 3.7

Elevation of base of Hydraulic Fill (m)QND

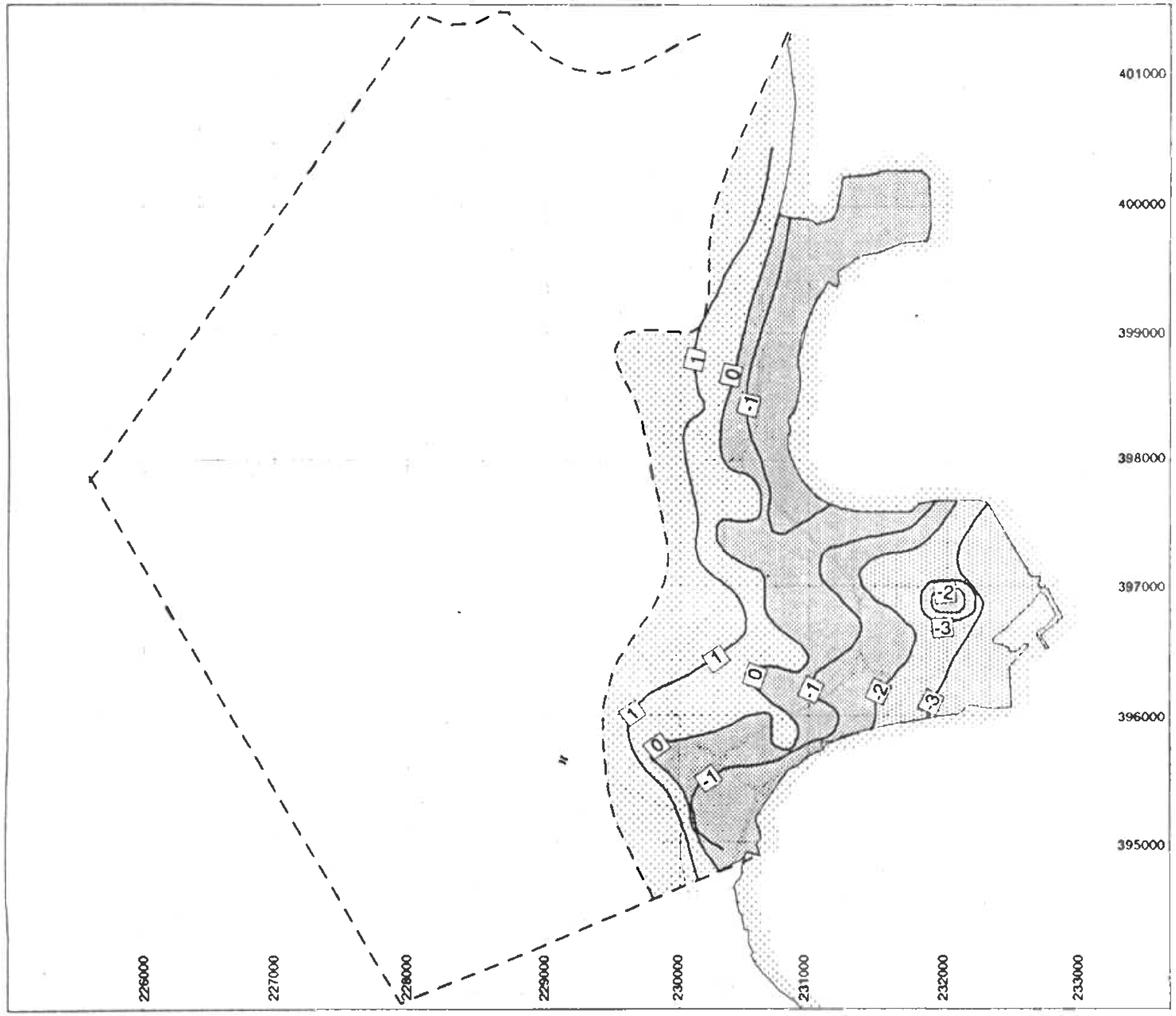


Figure 3.8

Thickness of Hydraulic Fill (m)

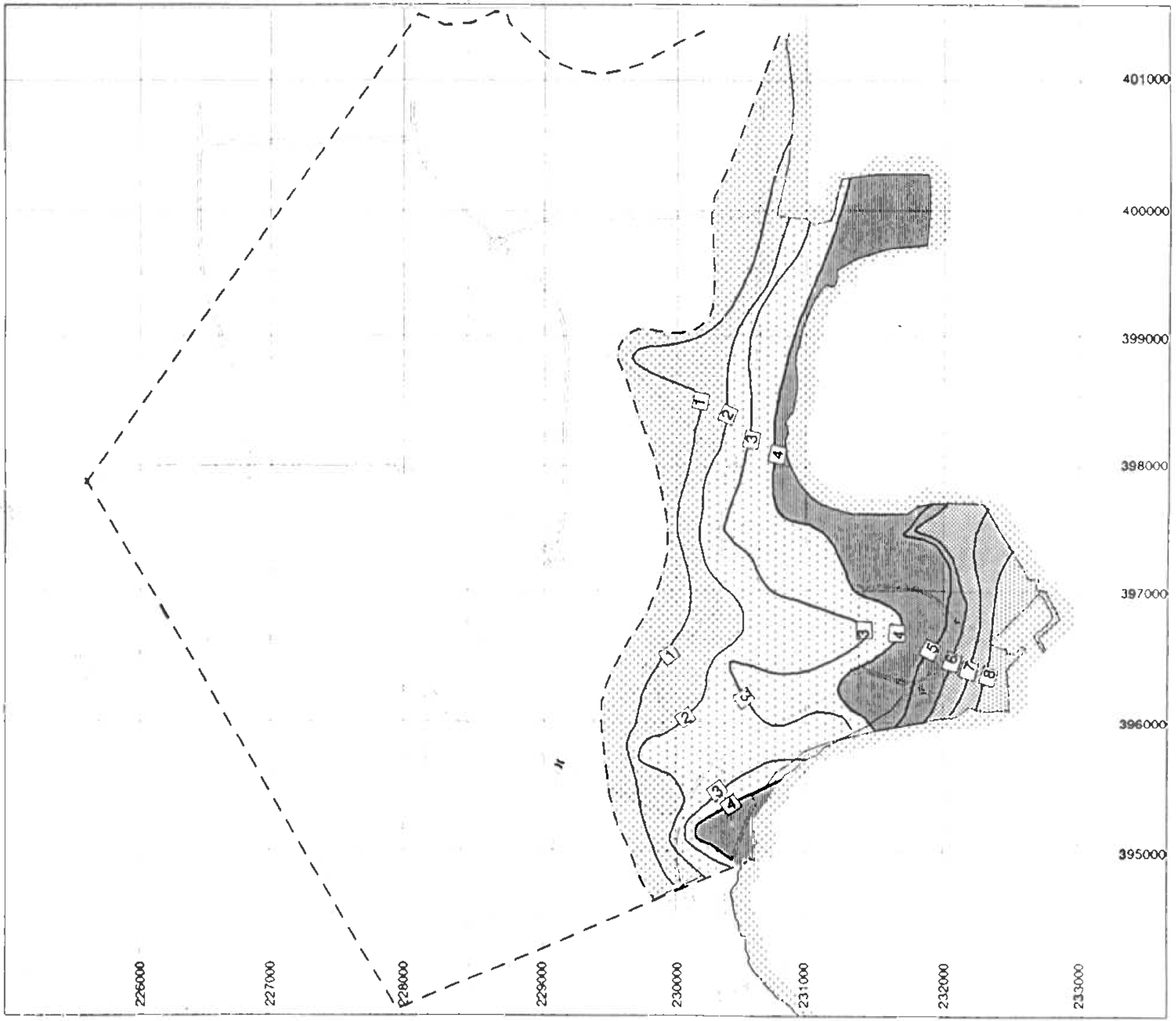


Figure 3.9

Elevation of Water Table (high tide 6/9/89) (m)

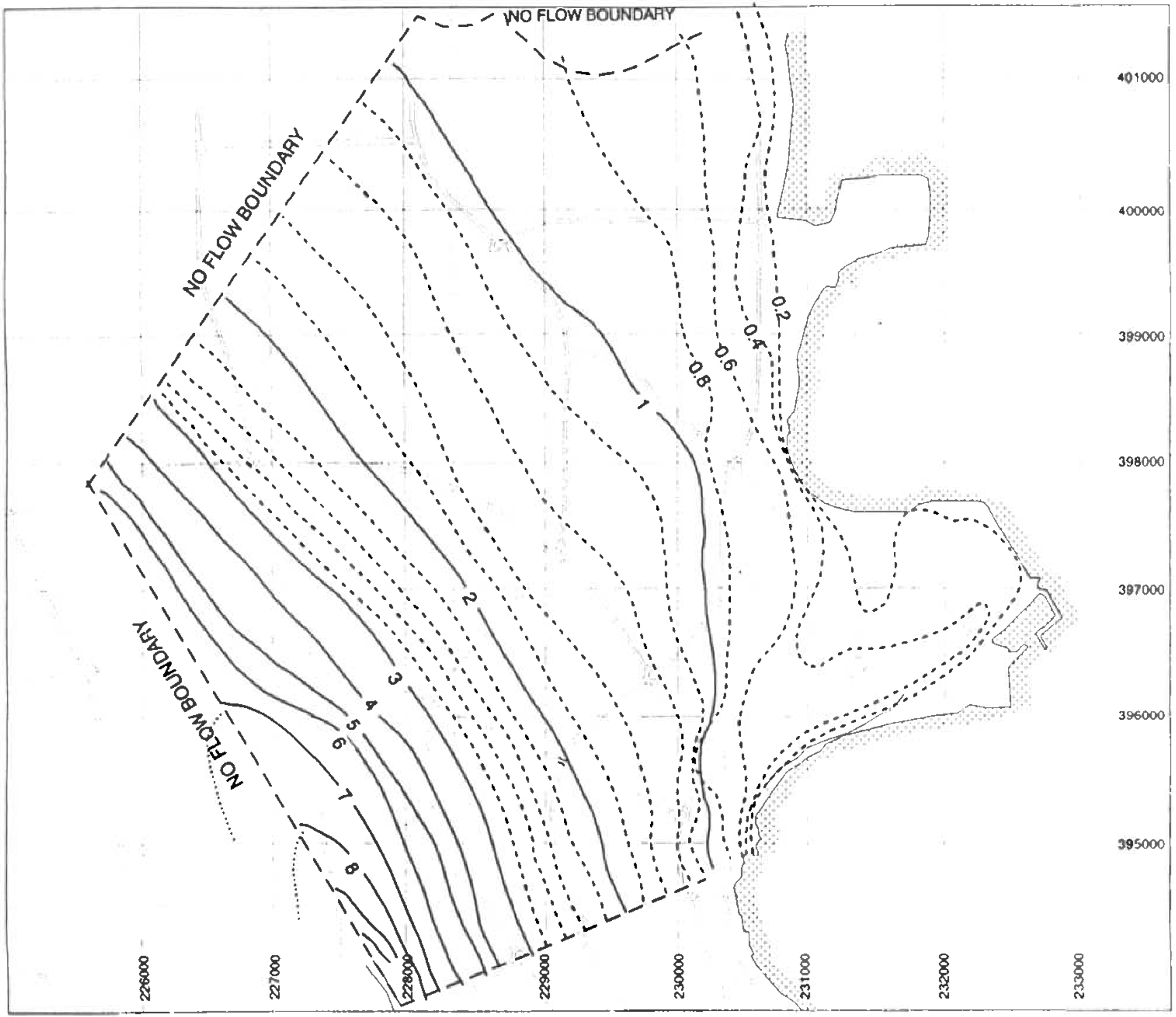


Figure 4.1

Depth to Water Table (high tide 6/9/89)(m)

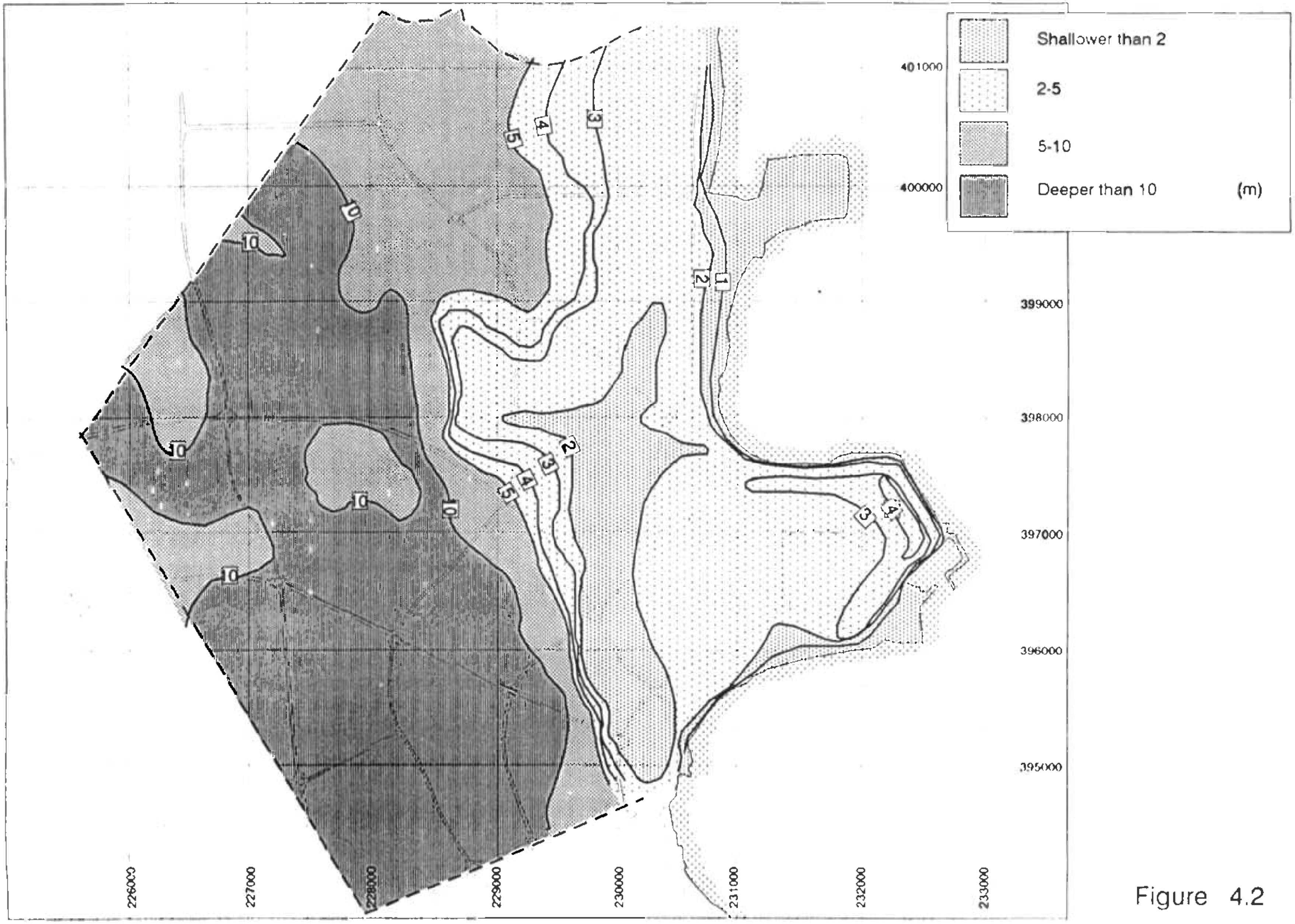
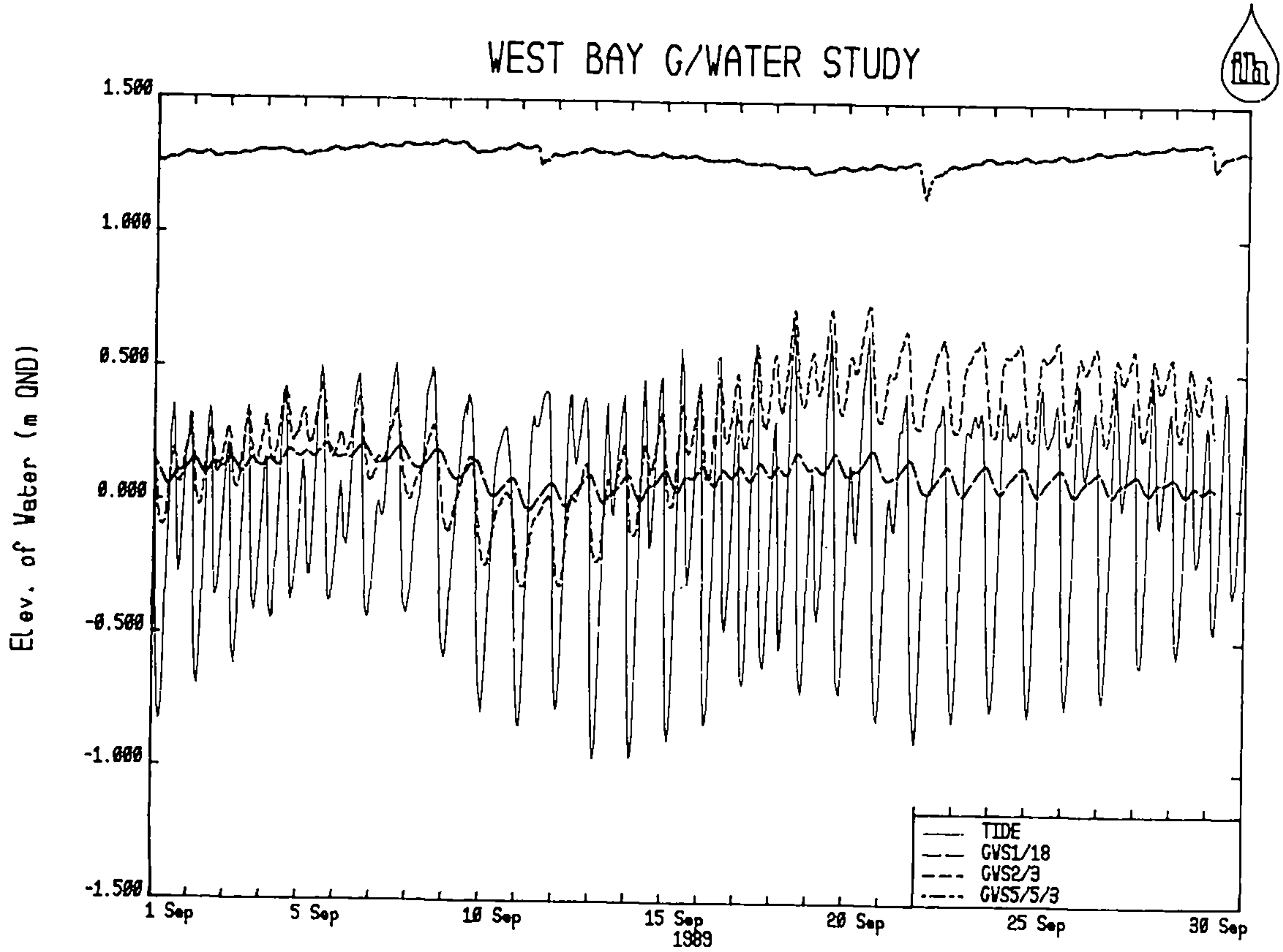


Figure 4.2



Tidal Responses in Boreholes

Figure 4.3

DOHA AIRPORT

Annual Rainfall

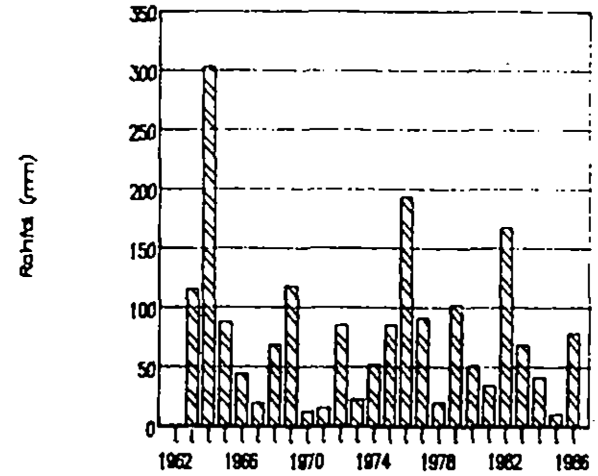


Figure 5.1

Mean Monthly Rainfall

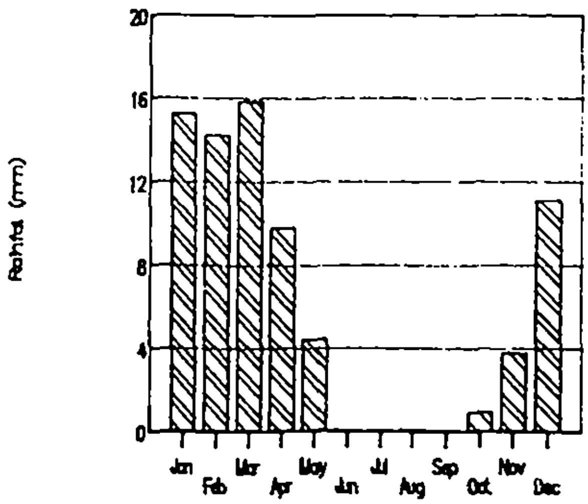


Figure 5.2

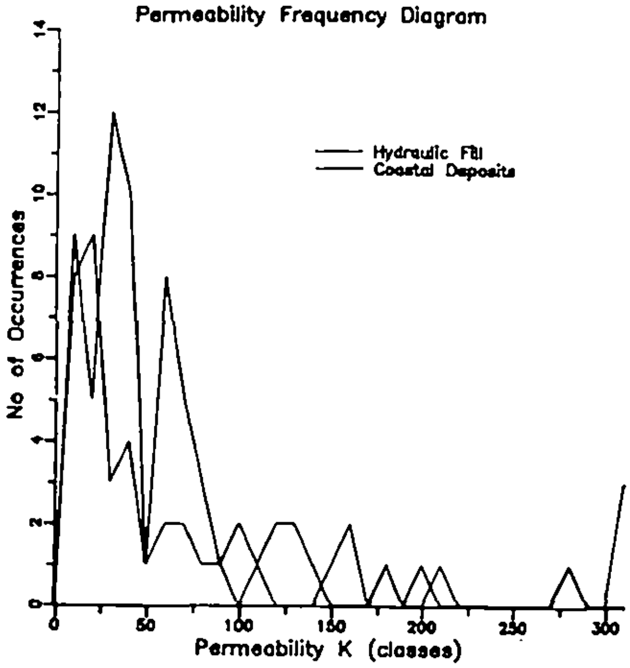


Figure 6.1

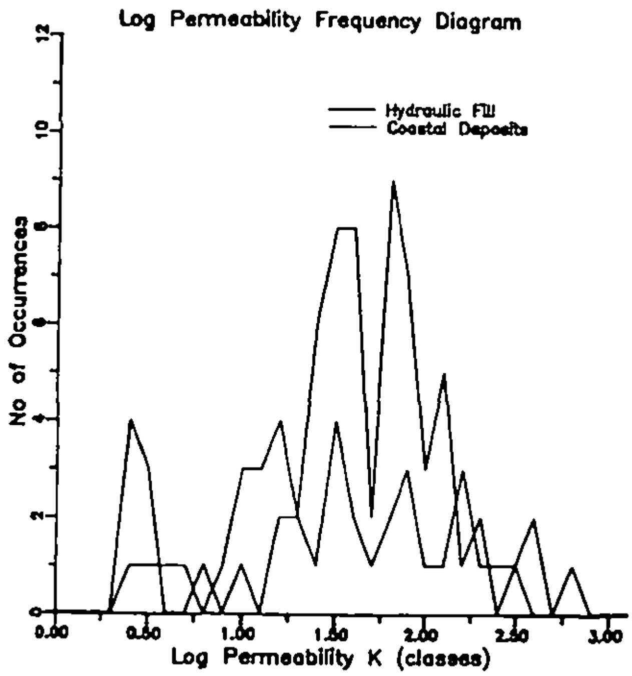


Figure 6.2

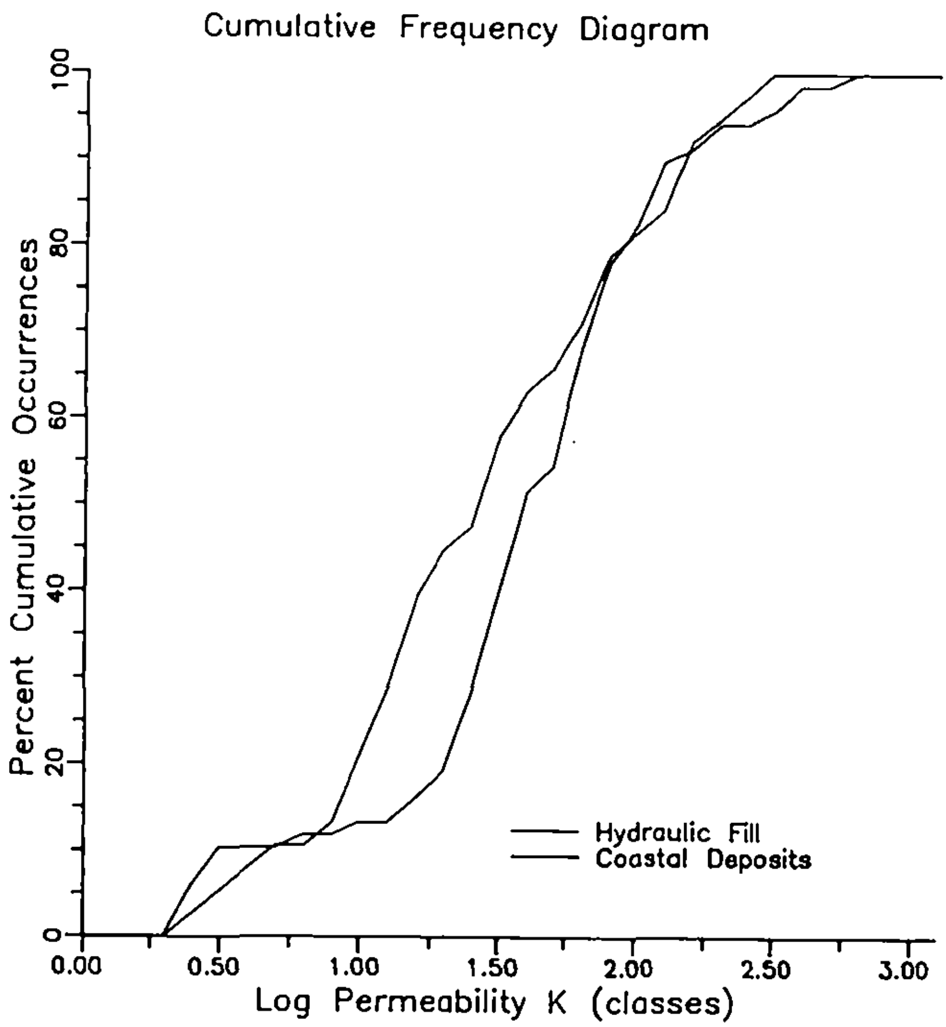

Cumulative Frequency Diagram
Percent Cumulative Occurrences
0
20
40
60
80
100
0.00
0.50
1.00
1.50
2.00
2.50
3.00
Log Permeability K (classes)
Hydraulic Fill
Coastal Deposits

Figure 6.3

Model-West Bay

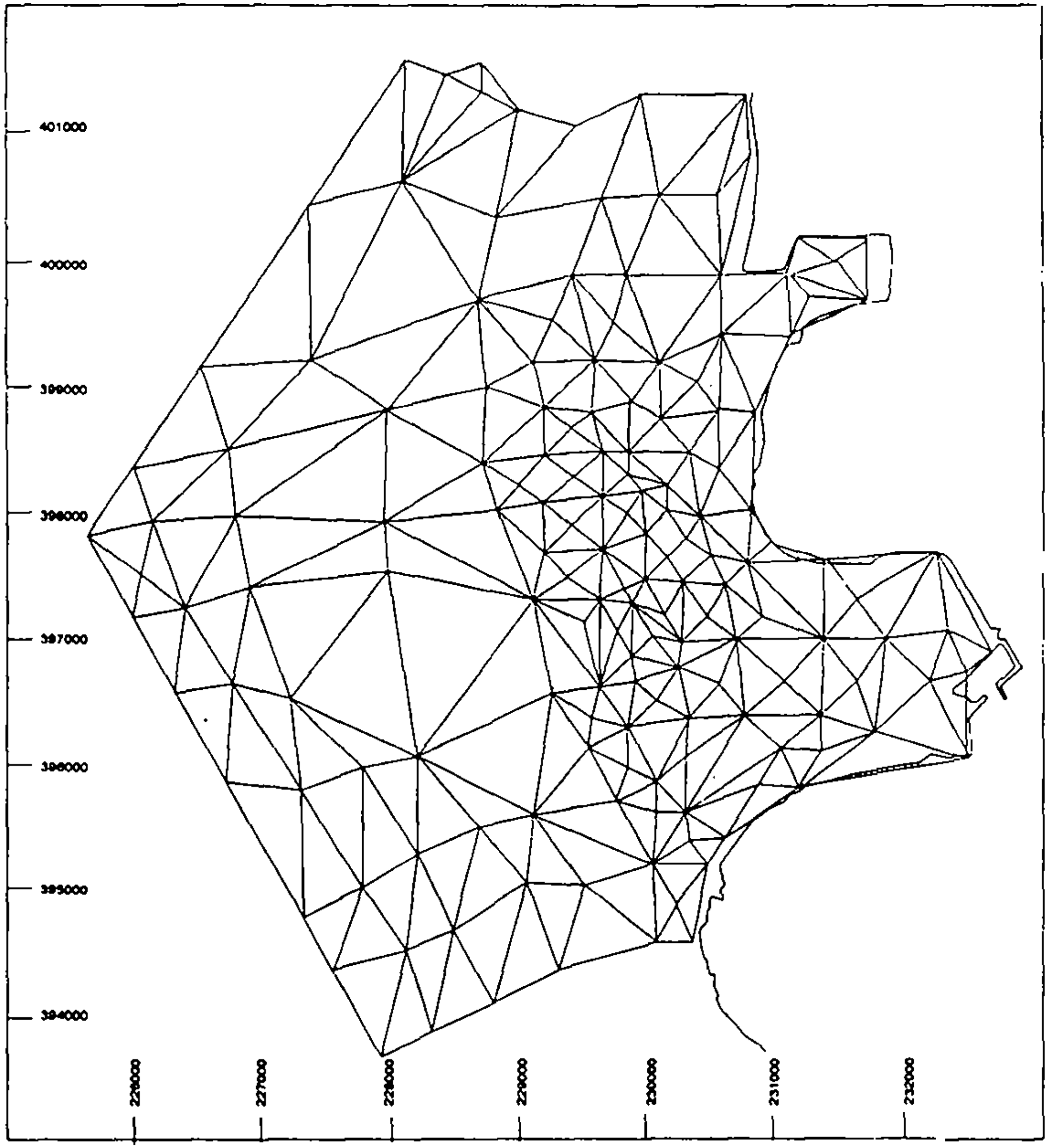


Figure 7.1

Appendix 1

APPENDIX 1

List of Engineering Reports relating to West Bay area

REPORT A: July 1989, Gulf Laboratories. Senior staff housing project, sub-surface site investigation. No. GD/199/L

REPORT B: January 1983, Wimpey Laboratories. Govt. of Iraq. Proposed new embassy in the new district of Doha. No. S/19872.

REPORT C: January 1982, Wimpey Laboratories. Gulf Organization Consulting Doha, Report on site investigation. No. S/18659.

REPORT D: May 1981, Wimpey Laboratories. Qatar General Petroleum Corporation. Proposed extension to headquarters building at new Doha. Report on site investigations. No.S/18003.

REPORT E: March 1983, Wimpey Laboratories. Proposed villa for H.E. Issa Al Kawari, New District of Doha. Report on site investigations. S/19931.

REPORT F: January 1983,Wimpey Laboratories. Government of Pakistan. Proposed new embassy of Pakistan in the New District of Doha. Report on site investigation. No. S/19673.

REPORT G: June 1984, Wimpey Laboratories. Islamic Republic of Iran. Proposed new embassy in the New District of Doha. Report on site investigation. No. S/21210.

REPORT H: February 1983, Wimpey Laboratories. Qatar General Insurance and Reinsurance Company. Proposed multi- storey office building in the New District of Doha. Report on site investigations. o. S/19923.

REPORT J: October 1982, Wimpey Laboratories. Qatar National Cement Company. Proposed headquarters building in the New District of Doha. Report on site investigation. No.S/19287.

REPORT K: September 1981, Wimpey Laboratories. Ministry of Works (ESD). Proposed offices for Ministry of Education, New District of Doha. Report on site investigation. No. S/18204.

REPORT L: April 1985, Wimpey Laboratories. Ministry of Public works. Qatar Sports Club. Site investigation report. No. S/16987/2.

REPORT M: October 1982, Wimpey Laboratories. Mannai Trading. Proposed Office and commercial Centre in the New district of Doha.Report on site investigation. No. S/19496.

REPORT N: March 1989, Gulf Laboratories. Qatar National Navigation and Transport Company. Proposed NNTC HQ - West Bay. Report on site investigation. No. GD/188/SI.

The demand for long-term scientific capabilities concerning the resources of the land and its freshwaters is rising sharply as the power of man to change his environment is growing, and with it the scale of his impact. Comprehensive research facilities (laboratories, field studies, computer modelling, instrumentation, remote sensing) are needed to provide solutions to the challenging problems of the modern world in its concern for appropriate and sympathetic management of the fragile systems of the land's surface.

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